

CHESSMAN DAM REHABILITATION INVESTIGATION

SUPPLEMENTAL REPORT

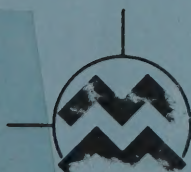
**GEOTECHNICAL AND PRELIMINARY
ENGINEERING**

SUBMITTED TO:

**CITY OF HELENA
PUBLIC WORKS DEPARTMENT
316 NORTH PARK
HELENA, MONTANA 59623**

DECEMBER 1985

PREPARED BY:



MORRISON-MAIERLE, INC.

CONSULTING ENGINEERS

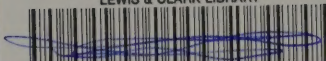
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PROJECT 500-107-02-(33)

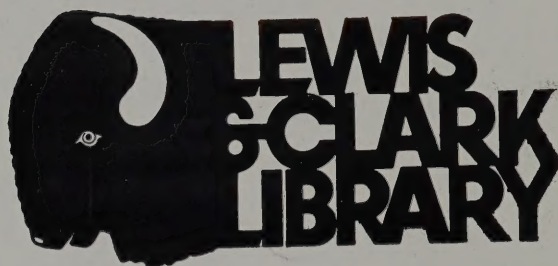
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CHESMAN DAM
REHABILITATION PLAN
SUPPLEMENTAL REPORT
DECEMBER 16, 1985

SUMMARY

1. Introduction

The City of Helena retained Harrison - Haeberle to conduct a rehabilitation investigation in June 1983. Harrison-Haeberle provided the following report and report presentation and Northern Engineering Inc. was under sub-contract, completed the stability analysis.

CHESMAN DAM
REHABILITATION PLAN

SUPPLEMENTAL DESIGN REPORT

DECEMBER 16, 1985

The Supplemental Investigation and Design Report was prepared by the City Commission on August 3, 1985.

2. Purpose

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The report contains the following information which is also part of the project and was prepared by the City Commission. The findings of the

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CHESSMAN DAM
REHABILITATION PLAN
SUPPLEMENTAL REPORT
DECEMBER 16, 1985

SUMMARY

A. Authorization

The City of Helena retained Morrison - Maierle to conduct a rehabilitation investigation of Chessman in June 1983. Morrison-Maierle provided the engineering analysis and report presentations and Northern Engineering and Testing under sub-contract, completed the geotechnical investigation and embankment stability analysis.

The supplemental investigation presented herein was authorized by the City Commission on August 5, 1985.

B. Purpose

In response to the findings of the potential unstable conditions at Chessman Dam in the initial stages of this investigation, the City drained the reservoir. The reservoir is not useful at this level as a raw water storage facility for the city's water supply. Because this primary storage facility is an integral component in the present planning of the City's Tenmile Water Supply System it was important to develop cost estimates for dam rehabilitation for consideration in the water system decision process. The purpose of this supplemental engineering investigation is to provide geotechnical data and engineering analyses sufficient for selecting a rehabilitation approach and for preparing a preliminary engineers estimate of the rehabilitation construction costs.

C. Background

Located about 12 miles southwest of Helena and constructed in the early 1900's the Chessman Dam has long been part of the Helena municipal water supply (Figure 1). Due to Chessman's location above the community of Rimini, the Main Dam was classified as a high hazard and was inspected under Phase I of the National Dam Safety Program in 1980. Similarly, the Saddle Dam which contains the spillway structure is also part of the Chessman project and was inspected in 1980. The findings of the Phase I dam safety inspection revealed certain deficiencies which required further investigation. In June 1983 Morrison-Maierle,

LOCATION MAP

FIGURE 1

CHESMAN DAM
REHABILITATION PLAN
SUPPLEMENTAL REPORT
DECEMBER 16, 1982

1. INTRODUCTION

A. Background

The City of Helena retained Morrison - Kierle to conduct a rehabilitation investigation of Chesman Dam in June 1981. Morrison-Kierle provided the engineering analysis and report presented to the City Council and the Montana Department of Transportation. The geotechnical investigation and assessment of the dam was completed.

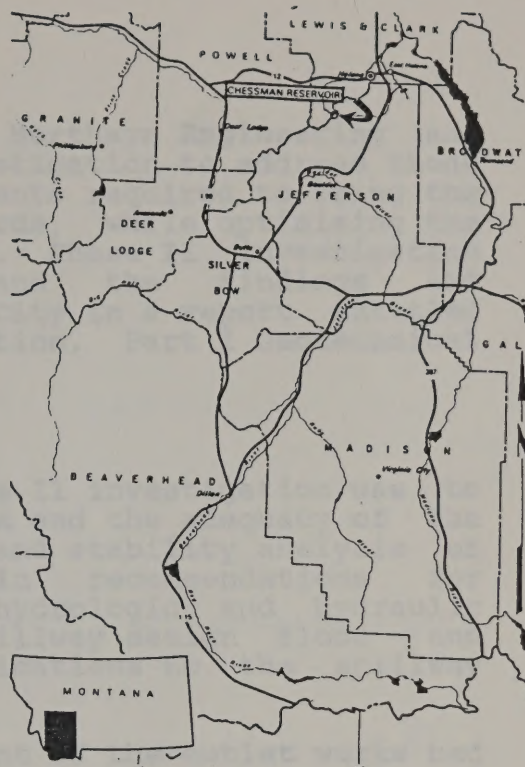
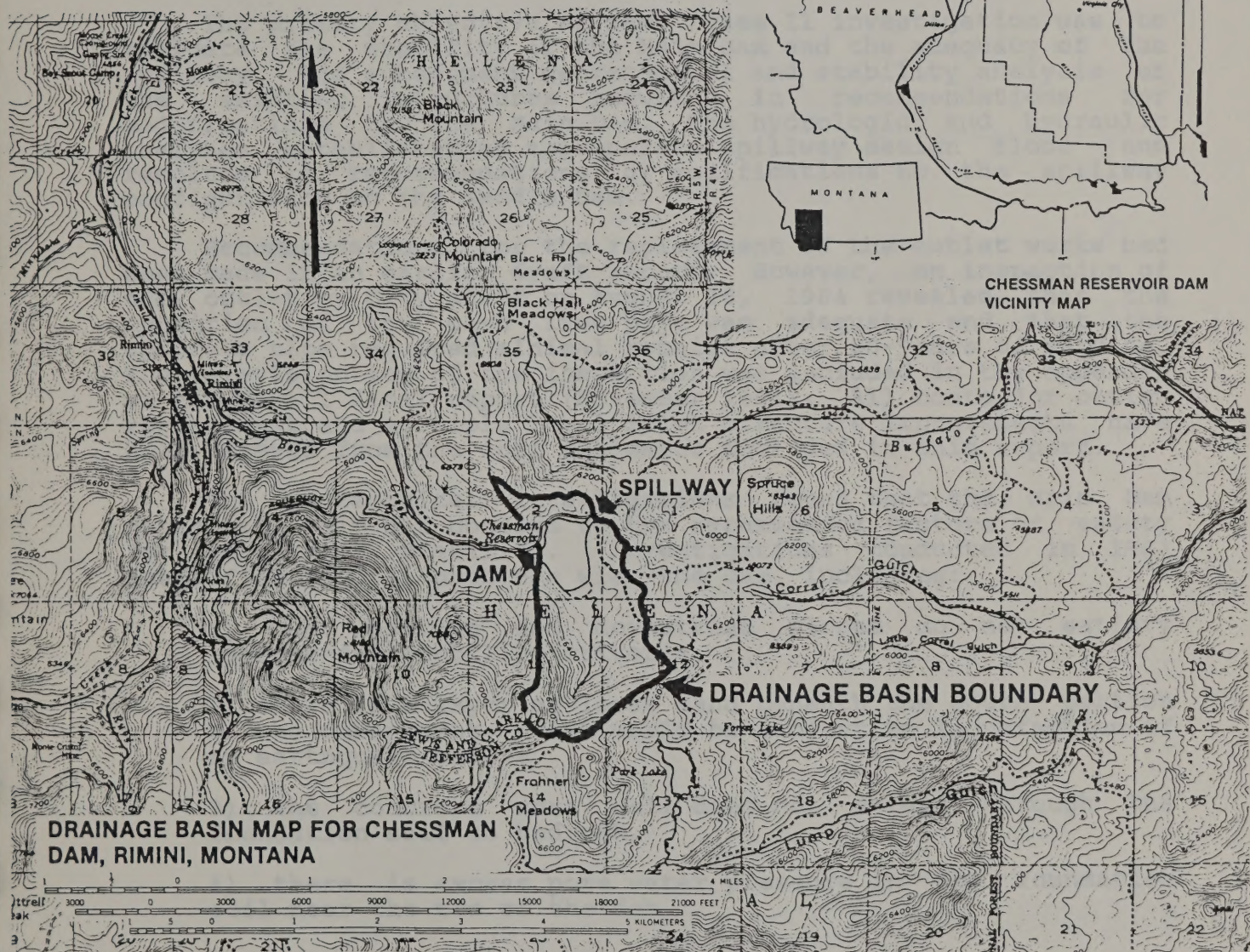
The supplemental investigation presented herein was authorized by the City Council on August 2, 1982.

B. Purpose

In response to the findings of the potential unstable conditions at Chesman Dam in the initial stages of this investigation, the City desired the reservoir. The reservoir is not needed at this level as a new water storage facility for the city's water supply. Because this primary storage facility is an integral component in the present planning of the City's Tonnage Water Supply System it was important to develop cost estimates for dam rehabilitation for consideration in the water system decision process. The purpose of this supplemental engineering investigation is to provide geotechnical data and engineering analyses sufficient for selecting a rehabilitation approach and for preparing a preliminary engineers estimate of the rehabilitation construction costs.

C. Description

Located about 15 miles southwest of Helena and constructed in the early 1900's the Chesman Dam has long been part of the Helena municipal water supply (Figure 1). Due to Chesman's location above the community of Helena, the Main Dam was classified as a high hazard and was inspected under Phase I of the National Dam Safety Program in 1980. Similarly, the Middle Dam which contains the spillway structure is also part of the Chesman project and was inspected in 1980. The findings of the Phase I dam safety inspection revealed certain deficiencies which required further investigation. In June 1982 Morrison-Kierle,



CHESSMAN DAM REHABILITATION INVESTIGATION LOCATION MAP

FIGURE 1



MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS

Inc., with geotechnical sub-consultant Northern Engineering and Testing, Inc., began a Phase II investigation to address those deficiencies and determine the improvements required to bring the facility up to a current safety standards, while optimizing the operation of the municipal water supply. Phase II investigation was completed in November 1984 and the findings and recommendations were submitted to the City in a report entitled "Chessman Dam Rehabilitation Investigation, Part I Geotechnical and Preliminary Engineering"

D. Phase II Investigation

The primary objective of the Phase II investigation was to evaluate the stability of the Main Dam and the adequacy of the spillway. The subsequent geotechnical and stability analysis of the existing structures resulted in recommendations for modifications to the Main Dam. The hydrologic and hydraulic analyses established an appropriate spillway design flood and resulted in recommendations for modifications to the spillway configuration on the Saddle Dam.

Recommendations for the replacement of the outlet works had been made in an earlier draft report. However, an inspection of the outlet by City staff on August 16, 1984 revealed that the condition of the cast iron pipe was adequate and that the concrete pipe required minimal repairs after 80-years of service. The television log of the inspection was included in the appendix of the Part I final report (November 1984). All costs for outlet replacement and multiple level inlet structure improvements have been removed from the cost estimates included in this report.

The geotechnical investigations found that the Main Dam embankment did not conform to the recommended factors of safety for embankment stability. Investigations conducted in 1983 identified several problems with the dam, including:

- 1) the upstream and downstream slopes do not satisfy recommended stability criteria,
- 2) there is a layer of loose sand beneath the south portion of the embankment, which would be subject to liquefaction if an earthquake occurs,
- 3) the concrete core wall does not extend through the foundation soil to bedrock,
- 4) there is excess pore water pressure in the foundation soil near the toe of the dam.

Recommendations for embankment stability included compaction grouting of a loose sand layer beneath the dam. Alternate 1 was to grout the entire sand layer upstream and downstream of the core wall to prevent liquefaction of this material during an earthquake. The extent of the loose sand was estimated from drill holes in the embankment taken as part of the Phase II evaluation. Alternate 2 was to grout only a portion of the loose sand layer

Two, with geotechnical sub-committee and Western Engineering and Testing, Inc., began a Phase II investigation to address those deficiencies and determine the improvements required to bring the facility up to a current safety standard. While conducting the operation of the municipal water supply, Phase II investigation was completed in November 1992 and the findings and recommendations were submitted to the City in a report entitled "Chesman Dam Rehabilitation Investigation, Part I Geotechnical and Preliminary Engineering".

2. Phase II Investigation

The primary objective of the Phase II investigation was to evaluate the stability of the dam and the adequacy of the existing structure. The assessment consisted of an analysis of the existing structure, including the foundation, abutments, and spillway. The analysis was based on the geotechnical and hydraulic data established in the previous investigation. The analysis also included an assessment of the existing structure for potential future improvements to the dam.

Recommendations for the rehabilitation of the dam were based on the results of the investigation. However, an investigation of the dam by City staff on August 14, 1992 revealed that the existing structure was in poor condition and that the dam was in need of major repairs. The investigation also revealed that the dam was in need of major repairs. The investigation also revealed that the dam was in need of major repairs. The investigation also revealed that the dam was in need of major repairs.

The geotechnical investigation found that the dam was in poor condition and that the dam was in need of major repairs. The investigation also revealed that the dam was in need of major repairs. The investigation also revealed that the dam was in need of major repairs. The investigation also revealed that the dam was in need of major repairs.

1) The upstream and downstream slopes do not satisfy recommended stability criteria.

2) There is a layer of loose sand beneath the south portion of the embankment, which would be subject to liquefaction if an earthquake occurs.

3) The concrete core wall does not extend through the foundation soil to bedrock.

4) There is excess pore water pressure in the foundation soil near the toe of the dam.

Recommendations for embankment stability included construction of a loose sand layer beneath the dam. Alternative 1 was to construct the entire sand layer upstream and downstream of the core wall to prevent liquefaction of this material during an earthquake. The extent of the loose sand was estimated from a 1991 study in the embankment taken as part of the Phase II investigation. Alternative 2 was to construct only a portion of the loose sand layer.

The following sketches illustrate the two alternates that have been under evaluation.



4

the Saddle Dam was \$825,000.

After the drilling had been completed and before the final recommendations had been made it became apparent that there were serious concerns on the safety of the Chessman Dam. With the potential for liquefaction of the Main Dam and inadequacy of the spillway in the Saddle Dam the City decided to drain the reservoir and await further recommendations. Meanwhile, an emergency warning plan was developed and placed into operation as a tool to be used in the event of a dam failure. This warning plan had been part of the earlier Phase I recommendations and was not entirely predicated by the knowledge of the stability problems at the Main Dam.

E. Supplemental Investigation

During the drought conditions which prevailed over much of Montana during the summer of 1985, the decision was made to continue the drilling program at Chessman. This supplemental drilling program determined more precisely the extent of loose sands both upstream and downstream of the core wall. The location of the loose sands has resulted in the final design recommendations and cost estimates for rehabilitation contained in this report.

Based upon drill hole information from the reservoir side on the upstream face of the dam, the extent of the loose sand layer was found to be greater than had originally been estimated. The possibility that the loose sand zone would reduce in size upstream of the core wall was dismissed by the information obtained from three scattered holes which produced data indicating an increasing zone of loose sand upstream of the core wall. This finding eliminated the choice of Alternate 1, as compaction grouting costs would be even greater than originally planned.

Limiting the rehabilitation plans to Alternate 2 and including updated cost information from specialty contractors dealing in compaction grouting and other construction fields, the new construction cost estimate for December 1985 is \$760,000.00

Detailed information regarding the supplemental engineering program and revised construction cost estimate is included in the following chapters. A summary of the recommendations made by this report are as follows:

TO RESTORE THE CHESSMAN PROJECT TO OPERATIONAL STATUS AND TO BE IN COMPLIANCE WITH CURRENT ENGINEERING GUIDELINES IT IS PROPOSED THAT THE FOLLOWING ITEMS BE CONSIDERED:

1. Rehabilitate the Main Dam by:

- a. flattening the upstream slope to 2-1/2 H to 1 V*
- b. widen the crest width to 45 feet*
- c. flatten the downstream slope to 3 H to 1 V*
- d. install pressure relief wells in the foundation*

- soils at the toe of the dam
- e. compaction grout the loose sands in the foundation and soils at the toe of the dam.
 - f. extend the concrete cutoff wall up to the top of the dam
 - g. extend the outlet works both upstream and downstream

2. Rehabilitate the Saddle Dam by:

- a. modifying the core wall
- b. widening the crest length of the exposed core wall
- c. providing an upstream and downstream slope transition of riprap

The estimated cost of the rehabilitation improvements is \$760,000.00

PERTINENT DATA
(revised April 1984)

CHESSMAN DAM, MT-1090; CHESSMAN SADDLE DAM, MT-1562

LOCATION: Township 8 North, Range 5 West
Section 2 latitude 112°11', longitude
46°28' Lewis and Clark County, Montana

WATERSHEDS: Beaver Creek, Buffalo Creek

OWNER/OPERATOR: City of Helena, Montana

PURPOSE: Municipal Water Supply

YEAR CONSTRUCTED: 1908

MAIN DAM: Earthfill dam with concrete core wall

SADDLE DAM: Earthfill dam with concrete core wall
Project spillway located in the dam

MAIN DAM CLASSIFICATION: Intermediate size, high downstream
hazard potential

SADDLE DAM CLASSIFICATION: Intermediate size, significant down-
stream hazard potential

KEY ELEVATIONS:

	(Project Datum)	(Revised City Datum)
Top of main dam	Elevation 105 feet	34 feet
Top of main dam core wall	101 feet	30 feet
Top of saddle dam and core wall	102 feet	31 feet
Top of causeway road berm	101 feet	30 feet
Maximum operating pool	100 feet	29 feet
Normal operating pool	99 feet	28 feet
Spillway Crest	97 feet	26 feet
Outlet at toe of main dam	65 feet	-- feet

MAIN DAM DIMENSIONS:

Hydraulic Height	40 feet from outlet
Structural Height	57 feet from bottom of core wall
Crest Length	440 feet
Crest Breadth	15-17 feet
Face Slopes	1V on 2H

PERTINENT DATA (Continued)

SADDLE DAM DIMENSIONS:

Hydraulic Height	17 feet from toe of chute
Structural Height	27 feet from bottom of core wall
Total Length	105 feet
Crest Breadth	10 feet, concrete paved
Face Slopes	Upstream 1V on 1.5H Downstream 1V on 2.5H

SPILLWAY:

(Saddle Dam)

Location	Left center of saddle dam
Type	Concrete-lined chute with broad crested weir control
Width	22.5 feet
Control	Provisions for flashboards to elevation 102 feet. Causeway road berm upstream at elev. 101 feet blocks spillway approach.
Crest Elevation	97 feet
Chute Slope	1V on 3H
Capacity at Overtopping	590 cfs, elevation 102 feet

OUTLET WORKS:

(Main Dam)

Type	Submerged 16-inch-diameter Cast Iron (CI) inlet pipe discharging to a 24-inch by 27-inch concrete arch conduit.
Length	100 feet CI pipe 88 feet concrete arch conduit
Control	Gate valve located in dry-well valve chamber at center of dam.
Capacity	Approximately 25 to 30 cfs at normal pool.

RESERVOIR:

Surface Area	114 acres @ Normal Pool
Storage at:	
Top of Main Dam	2370 acre-feet @ elev. 105 feet
Top of Saddle Dam	1990 acre-feet @ elev. 102 feet
Normal Pool	1630 acre-feet @ elev. 99 feet
Spillway Crest	1430 acre-feet @ elev. 97 feet

CHESS3/E

1. DAM SAFETY CRITERIA

The 1980 inspection and evaluation of the Chessman Dam and Chessman Saddle Dam under the National Dam Safety Program revealed certain deficiencies in the project which resulted in recommendations for further investigations. Phase II investigations commenced in 1983 address those deficiencies and determine the necessary requirements to provide a safe facility while optimizing the operation of this project.

The primary emphasis of the investigations completed thus far, addressed the:

- * *stability of the Main Dam embankment,*
- * *adequacy of the spillway within the Saddle Dam,*
- * *development of a downstream warning plan.*

The geotechnical and stability analyses of the existing structure resulted in recommendations for modifications to the Main Dam embankment. The hydrologic and hydraulic analyses established a spillway design flood and resulted in the recommendations for modifications to the Saddle Dam.

The geotechnical investigation concluded that the Main Dam embankment did not meet the recommended factors of safety at various design conditions, including static conditions (without earthquake), seismic conditions, and rapid drawdown. Of these the seismic condition indicated a critical condition existed due to a loose sand layer being present in the embankment.

The results of the slope stability analyses for the existing Main Dam embankment were as follows:

	Calculated	Recommended
	Factors of Safety	
Reservoir Condition		
Reservoir Full (elevation 99) Steady State Seepage without earthquake	1.1	1.5
Reservoir Full (elevation 99) Steady State Seepage with earthquake	0.7	1.0
Reservoir Full Sudden drawdown	1.0	1.2

2. GEOTECHNICAL INVESTIGATION

In compliance with recommendations from the Phase I dam safety inspection geotechnical investigations at Chessman were conducted in 1983. In a December 1, 1983 report, Northern Engineering and Testing, Inc. (NET) identified two primary deficiencies with the stability of the upstream and downstream slopes of the Main Dam: 1) potential liquefaction of the loose sand layer beneath the dam during an earthquake and, 2) high pore water pressure near the toe of the dam.

The most feasible solutions for upgrading the dam's stability and correcting the deficiencies included two plans for using compaction grouting on the loose sand layer. One plan involved compaction grouting both upstream and downstream of the core wall while the other plan involved only the downstream portion where more data was available.

To accurately evaluate rehabilitation alternatives and to provide data for design at a later date, a supplemental field investigation was authorized in August 1985 to obtain additional information on the loose sand layer. The supplemental investigation included:

- * Definition of the extent of the loose sand layer below the embankment by performing test borings on the upstream and downstream face of the dam. Investigation determined the relative feasibility of Alternates 1 and 2, as they were discussed in the Final Report Geotechnical and Preliminary Engineering - November 1984.

- * Investigation to identify a borrow source with a minimum of 20,000 cubic yards of common borrow and a suitable source of riprap.

- * Performance of the necessary geotechnical analysis to support recommendations for final design and cost estimates for the most attractive rehabilitation alternate.

The results of NET supplemental investigation were presented in: "Report of Geotechnical Investigation Phase II - Chessman Dam Helena, Montana" dated November 27, 1985. Following is a summary of the geotechnical findings. The text of the geotechnical report is included in the APPENDIX. Laboratory test information of all data is available in the NET report.

A. Embankment Area

Additional test borings made on the upstream slope of the embankment indicate the loose sand layer is more extensive in this area than previously anticipated. The approximate revised limits of the loose sand layer and the location of all drill holes are shown on Figure 2. The elevation of the bottom of the loose sand layer is somewhat variable, but based on the test borings, it appears that overexcavation to approximately elevation 60, within the limits shown should remove the layer.

B. Borrow Area

An area in the northeast corner of the reservoir was investigated as a borrow source. The location of the borrow area and test pits are shown on Figure 3. In this area, clayey or silty sand overlies granite bedrock. The depth to bedrock ranges from about 7 to more than 10 feet. It appears there is in excess of 20,000 cubic yards of sand available in this area.

Laboratory tests indicate the sand has physical and engineering properties similar to the material in the existing embankment. Triaxial shear tests indicate an angle of internal friction of about 36 degrees for remolded samples of sand.

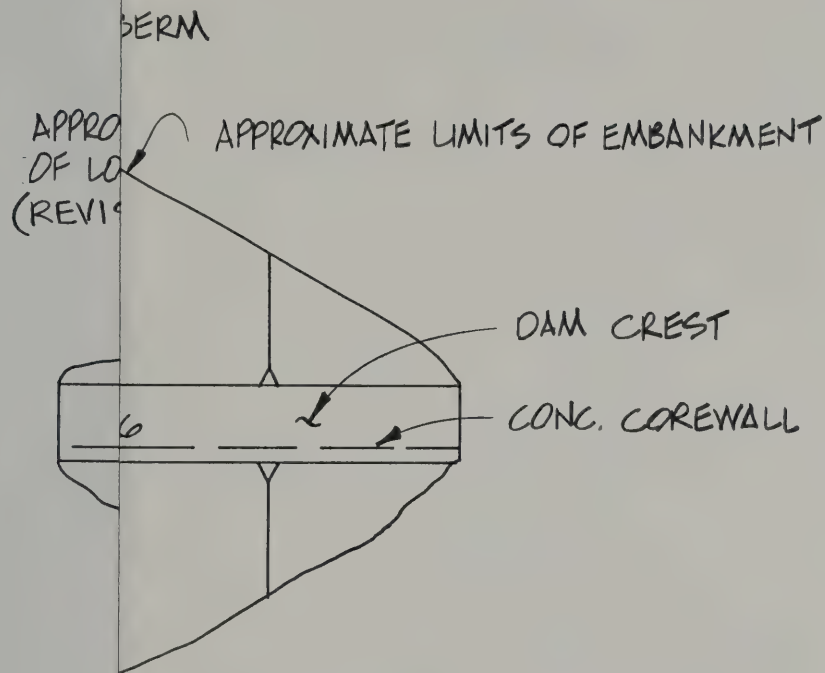
Water was encountered in many of the test pits. The in-place moisture content of the sand is above optimum moisture and unless the reservoir level is drawn down further, it will probably be necessary to dry the borrow in order to place and compact it.

C. Riprap Source

The existing riprap on the upstream slope of the dam is in good condition and appears to have performed well. It consists of near cubical shaped blocks and was placed by hand to achieve a smooth face. The existing riprap can be removed and reused.

Two additional sources of riprap were investigated. The most promising source appears to be an old mine adit, approximately 2000 feet northwest of the dam. There are several piles of quartz monzonite boulders near the mine adit. Most of the boulders are approximately 1 to 1 1/2 feet in diameter, and do not appear to have weathered appreciably. This may be a source of the riprap used in the original construction.

A talus slope, approximately one mile southwest of the dam was also investigated. Material in the slope consists of angular rhyolite particles, ranging in size from 2 inches to about 2 feet. There is a large quantity of talus present, but visual observation indicates it may be subject to weathering. If this source is considered, durability tests should be conducted in the laboratory to determine if the quality is acceptable.



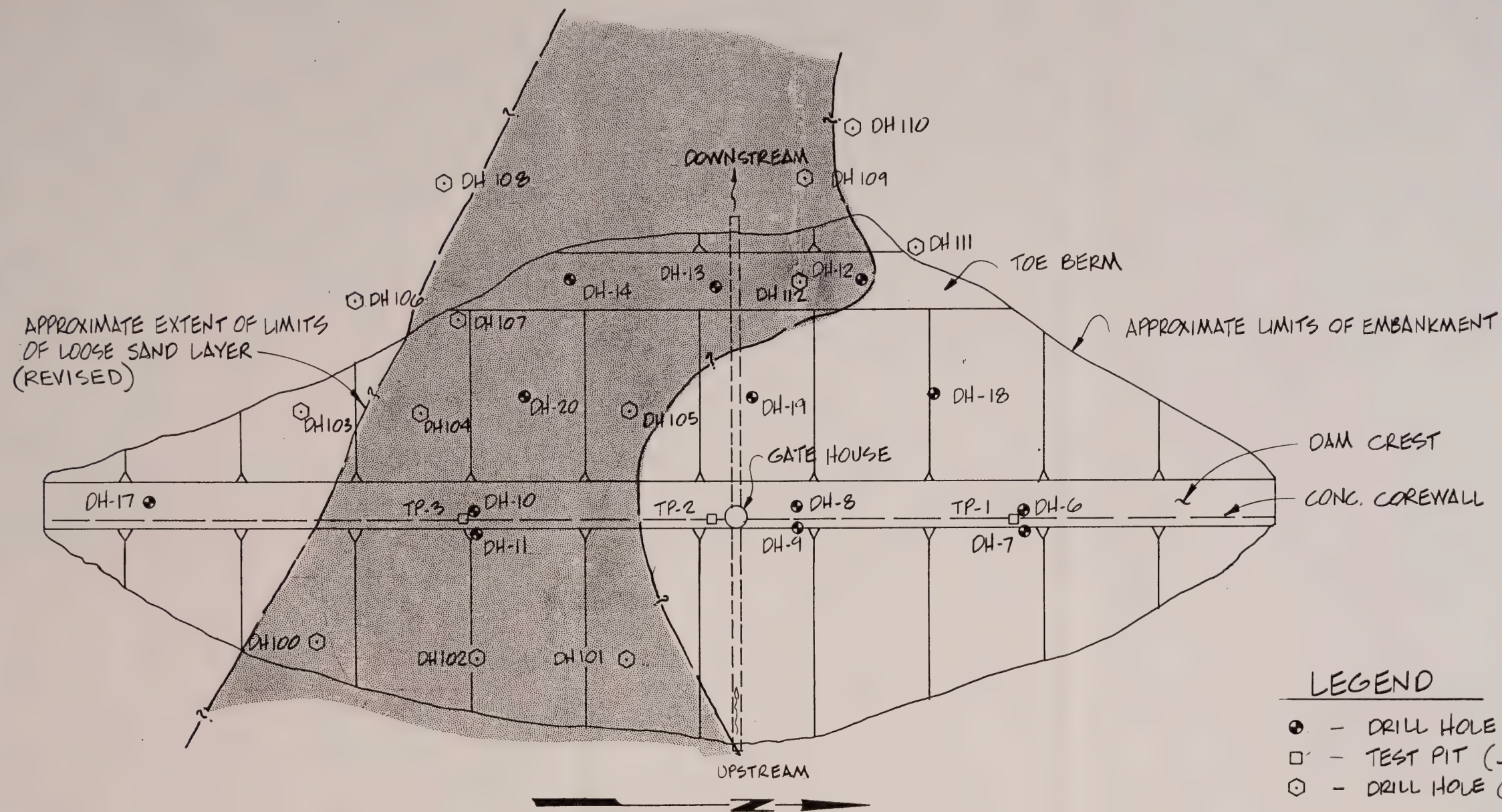
LEGEND

- ⊕ - DRILL HOLE (JULY-OCT. 1983)
- - TEST PIT (JULY 1983)
- ⊙ - DRILL HOLE (SEPT. 1985)

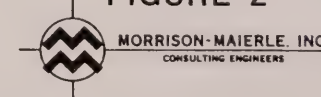
CHESSMAN DAM
REHABILITATION INVESTIGATION
REVISED LIMITS OF LOOSE SAND
&
LOCATION OF DRILL HOLES
FIGURE 2



MORRISON-MAIERLE, INC.
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CHESSMAN DAM
 REHABILITATION INVESTIGATION
 REVISED LIMITS OF LOOSE SAND
 &
 LOCATION OF DRILL HOLES
 FIGURE 2







● - TEST PITS

CHESSMAN DAM
REHABILITATION INVESTIGATION
TEST PITS - BORROW
AND RIPRAP SOURCES
FIGURE 3



MORRISON-MAIERLE, INC.
 CONSULTING ENGINEERS

1" = 500'

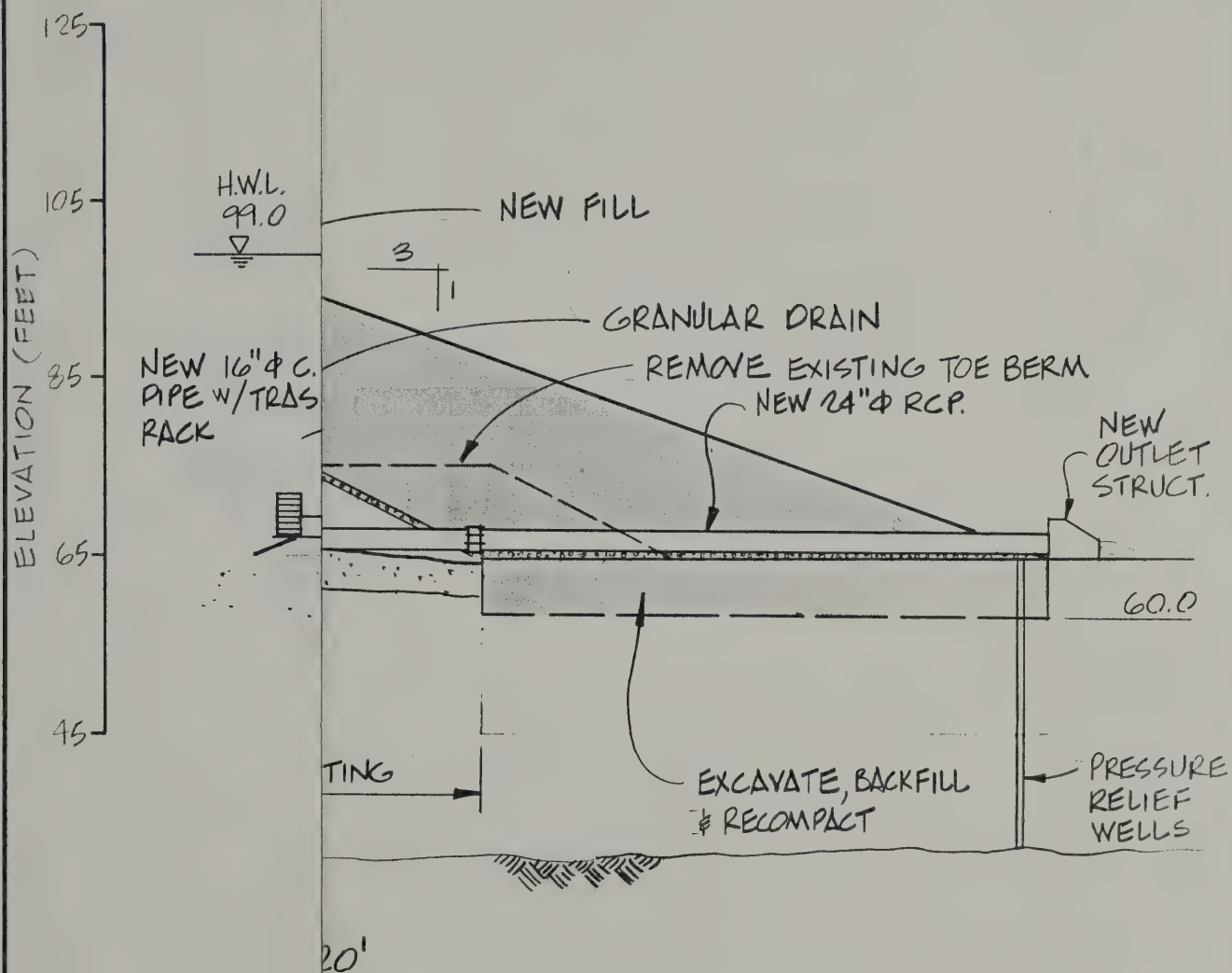
3. PRELIMINARY DESIGN RECOMMENDATIONS

Alternates considered in the rehabilitation plan were described in previous sections and in prior reports. Alternate 1 involved densifying the loose sand layer beneath the entire embankment. Since the loose sand layer is now known to be more extensive than anticipated beneath the upstream portion of the embankment, this alternate is no longer the most cost effective.

Alternate 2, the recommended alternate, involves densifying the loose sand layer downstream of the concrete cutoff. The crest of the embankment would be widened by 30 feet. If the sand layer upstream of the corewall liquefies and causes embankment failure, with this system there would still be sufficient embankment present to prevent the loss of the reservoir. As part of this alternate, the upstream and downstream slopes of the embankment would be flattened, an internal drain would be constructed in the downstream embankment, and pressure relief wells would be installed near the toe of the embankment. The plan is illustrated on Figure 4. The tasks required to stabilize the Main Dam are as follows:

1. Densify the loose sand layer downstream of the concrete core wall by overexcavating and replacing it where possible and densifying the remaining embankment in-place through compaction grouting techniques.
2. Construct a granular drain on the downstream slope.
3. Place fill to widen the crest by 30 feet and slope the downstream fill at 3 horizontal to 1 vertical.
4. Install pressure relief wells around the toe of the embankment.
5. Place fill and flatten the upstream slope to 2 1/2 horizontal to 1 vertical.

A major component of the rehabilitation plan is the densification of the loose sand layer. The recommendations have indicated that where possible densification should be by overexcavation and replacement with compacted fill. Maximum excavation slopes should be no steeper than 1 horizontal to 1 vertical above the water table and 2 horizontal to 1 vertical below the water table. This procedure would provide the most economical improvement. Overexcavation was considered in Alternate 2 as illustrated on Figures 4 & 5.



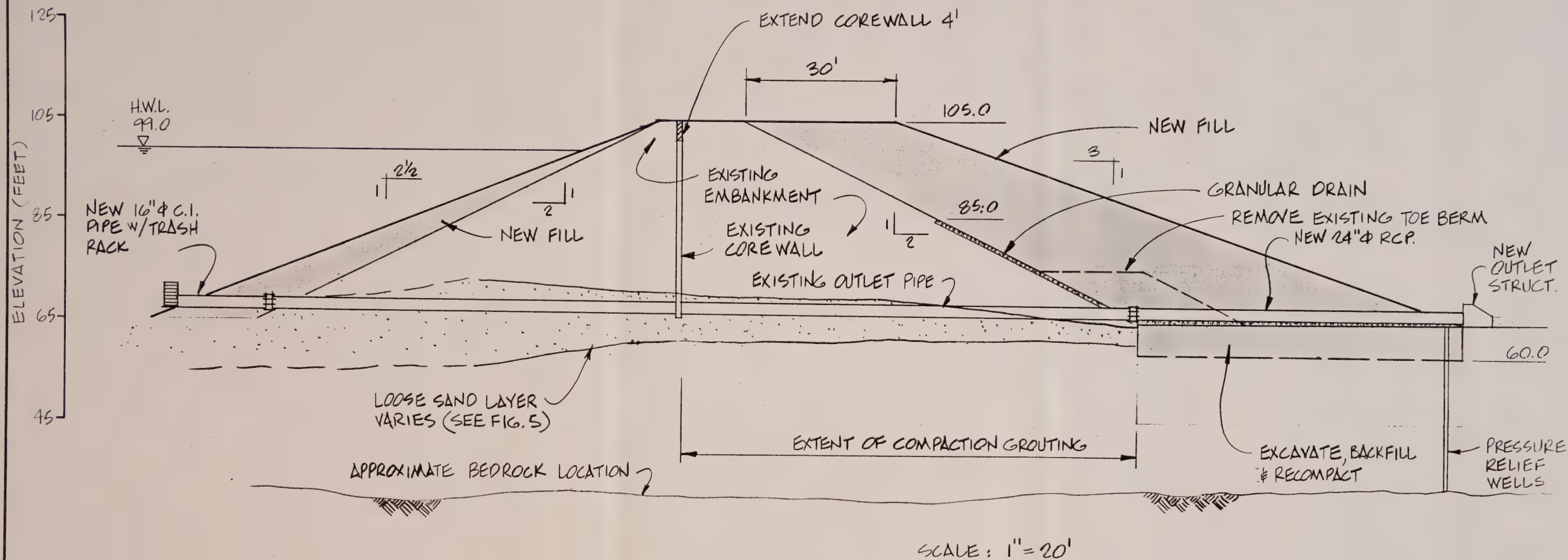
CHESSMAN DAM
 REHABILITATION INVESTIGATION

CROSS SECTION
 MAIN DAM REHABILITATION

FIGURE 4



MORRISON-MAIERLE, INC.
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MAIN DAM CROSS SECTION ALONG
OUTLET WORKS

CHESSMAN DAM
REHABILITATION INVESTIGATION

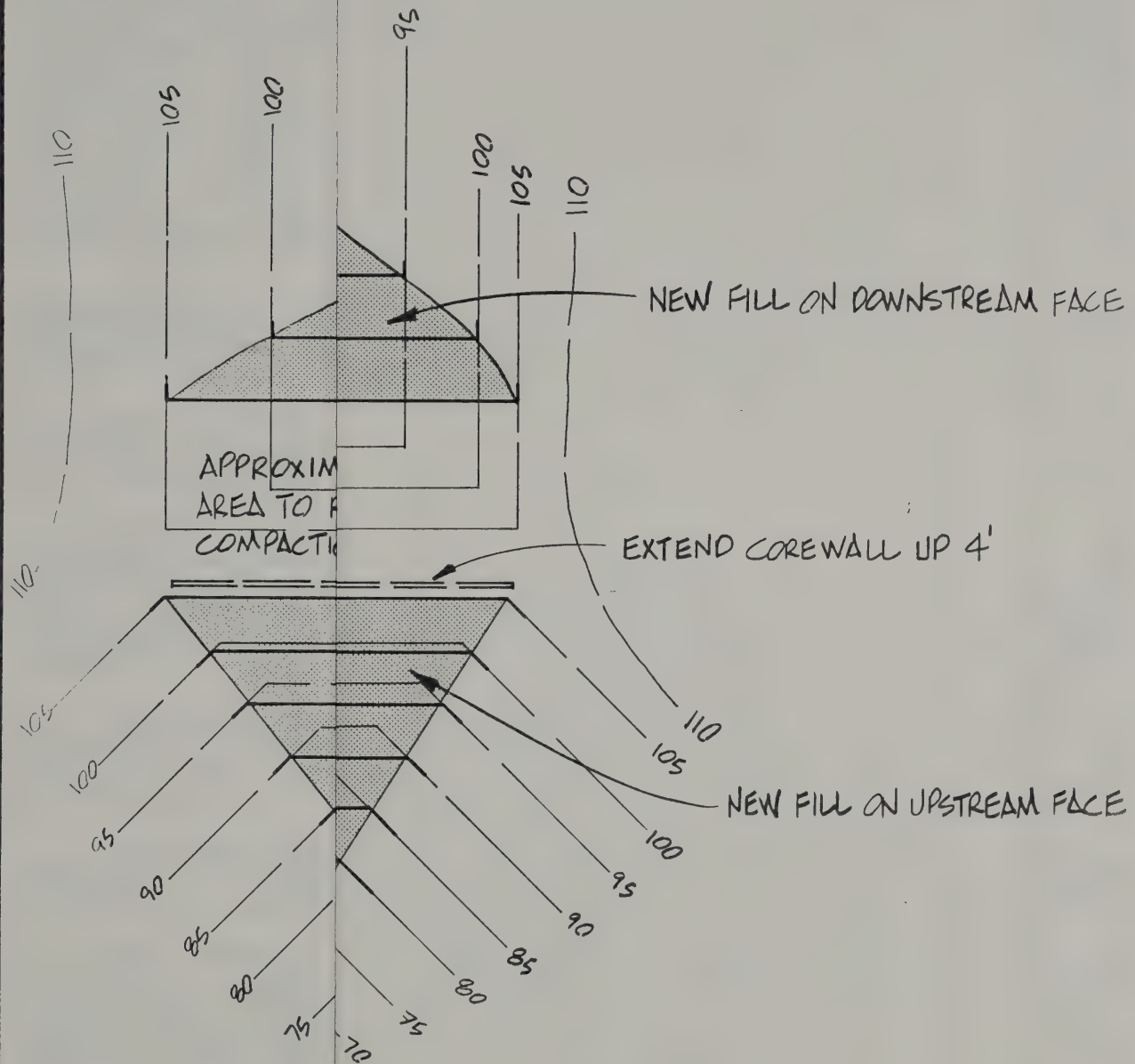
CROSS SECTION
MAIN DAM REHABILITATION

FIGURE 4



MORRISON-MAIERLE, INC.
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EXCAVATED TO ELEV.
ED & RECOMPACTED



**CHESSMAN DAM
REHABILITATION INVESTIGATION**

**PLAN VIEW
MAIN DAM REHABILITATION**

FIGURE 5



MORRISON-MAIERLE, INC.
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PRESSURE RELIEF WELLS
7 TYP.

THIS AREA TO BE EXCAVATED TO ELEV.
60.0 & BACKFILLED & RECOMPACTED

NEW 24" RCP
EXT. W/OUTLET STRUCT.

NEW FILL ON DOWNSTREAM FACE

APPROXIMATE LIMITS OF
AREA TO RECEIVE
COMPACTION GROUTING

EXISTING OUTLET WORKS

EXTEND COREWALL UP 4'

NEW FILL ON UPSTREAM FACE

NEW 16" C.I. EXTENSION
W/TRASH RACK

RESERVOIR



SCALE: 1" = 40'

**CHESSMAN DAM
REHABILITATION INVESTIGATION**

**PLAN VIEW
MAIN DAM REHABILITATION**

FIGURE 5



MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS

Various methods of densifying in-place materials were analyzed for use at Chessman. Dynamic compaction, vibroflotation and compaction grouting are all techniques of providing in-place densification. Evaluation of techniques indicated that both dynamic compaction and vibroflotation caused settlement which could damage the core wall or outlet works. Additionally, vibroflotation may not be effective due to the clay content in the embankment soils. Compaction grouting appeared to be the most favorable means of densifying the sand layer. Approximate costs for this procedure were obtained from the Hayward - Baker Company, Ground Modification Division, a company experienced in performing compaction grouting, and are included in the construction cost estimate.

A granular drain between the existing embankment and the new downstream fill should consist of clean, free draining aggregate, and should be protected from contamination and plugging with a filter fabric.

The upstream slope of the embankment should be flattened to increase the stability under rapid drawdown conditions. The use of a toe berm was considered but does not appear feasible.

Riprap on the flattened upstream slope of the embankment is necessary to prevent erosion from wave action. Existing rock can be reused and additional sources are located close by. A filter fabric may be used as a substitute for a gravel layer between the embankment slope and riprap to prevent piping of fine material through the rock.

Construction of a toe drain was considered to relieve excess hydrostatic pressure in the foundation soils beneath the toe of the dam but installation of relief wells prove to be more economical.

The extension of the core wall to the top of the dam and the extension of the outlet works both upstream and downstream, remains part of these recommendations.

The Saddle Dam recommendations remain unchanged from previous reports. The modifications generally include establishing the core wall crest at elevation 99, increasing the crest length to 105 feet, and providing riprap protection on all exposed slopes to withstand overtopping failure.

4. CONSTRUCTION COST ESTIMATE

The revised construction cost estimate contained Table 1 indicates updated information from the November 1984 submittal on the following asterisked items:

- * Item 1: Compaction grouting --
Reduced the total number of holes, and
quantity of grout based upon NET data
- * Item 2: Subexcavation, compact foundation materials --
Reduced the quantity of materials to be
removed and compacted based on NET information
- * Item 3: Place and compact downstream fill material --
Increased unit price to \$5.00/CY
- * Item 5: Drain Material -- Reduced the quantity of
drain material based on recommendation from
NET
Increased unit cost to \$10.00/CY
- * Item 6: Filter fabric -- Increased unit cost to
\$3.00/SY
- * Item 7: Remove and stockpile riprap -- Reduced
quantity based on observed information from
NET
- * Item 8: Place and compact upstream fill material --
Increased unit cost to \$5.00/CY
- * Item 9: Furnish and place riprap --
Reduced quantity based upon recommendation
from NET regarding thickness
Increased unit cost to \$28.00/CY to include
cost of filter fabric in lieu of graded
blanket material
- * Item 10: 6 inch diameter vertical wells --
Reduced quantity based upon NET recommendation
- * Item 11: Grade and reseed slopes -- Increased unit
cost to \$2000/ACRE
- * Item 23: Membrane lining -- Increased unit cost to
\$3.00/SY
- * Item 24: Filter blanket -- Reduced unit cost to
\$5.00/CY to reflect use of native materials
- * MOBILIZATION modified to included all mobilization
except that of the specialty contractor
performing the compaction grouting, which is
included as Item 1c.

* GEOTECHNICAL / CONSTRUCTION MANAGEMENT SERVICES
 Reduced to reflect the effort expended by
 this supplemental engineering and
 geotechnical effort to define the loose sand
 layer and determine parameters for final
 design of embankment, drain and riprap
 materials.

Also indicated on the construction cost estimate is a
 projection of costs into 1986 and 1987. These include inflation
 factors of 3.0 percent for 1986 and 6.0 percent for 1987.
 Table 1.

Revised costs for Alternate 1 were developed for comparison
 with Alternate 2 and were estimated from information included in
 the November 1984 report and supplemental data gathered for this
 report. The cost comparison between Alternates 1 and 2 is
 summarized below:

	Alternate 1	December 1985.....	\$850,000.00
Recommended:	Alternate 2.....	December 1985.....	\$760,000.00

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TABLE 1.
CONSTRUCTION COST ESTIMATE CHESSMAN MAIN DAM REHABILITATION ALTERNATE 2

ITEM	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	AMOUNT	
****	*****	*****	****	*****	*****	
MAIN DAM						
1	Compaction grouting				\$167,450.00	* Denotes change
1a	Drilling	4,200	LF	\$16.00		from November 1984
1b	Grouting	145	CY	\$450.00		
1c	Monitoring - Mob/demobilization	1	LS	\$35,000.00		
2	Subexcavate, compact foundation	4,830	CY	\$6.00	\$28,980.00	*
3	Place and compact downstream face	19,820	CY	\$5.00	\$99,100.00	*
4	Clear and scarify slopes	1	EA	\$3,200.00	\$3,200.00	
5	Drain material	1,560	CY	\$10.00	\$15,600.00	*
6	Filter fabric	5,070	SY	\$3.00	\$15,210.00	*
7	Remove and stockpile riprap	1,000	CY	\$6.00	\$6,000.00	*
8	Place and compact upstream fill	4,960	CY	\$5.00	\$24,800.00	*
9	Furnish and place riprap with blanket	1,850	CY	\$28.00	\$51,800.00	*
10	6 inch diameter vertical wells	140	LF	\$15.00	\$2,100.00	*
11	Grade and reseed slopes	1	ACRE	\$2,000.00	\$2,000.00	*
EXTEND OUTLET WORKS						
12	Furnish and install 16" cast iron pipe	25	LF	\$40.00	\$1,000.00	
13	Furnish and install 24" RCP	70	LF	\$40.00	\$2,800.00	
14	Outlet basin	5	CY	\$500.00	\$2,500.00	
15	Inlet structure with trashrack	1	EA	\$2,000.00	\$2,000.00	
EXTENSION OF CORE WALL						
16	Excavate and stockpile embankment	960	CY	\$4.00	\$3,840.00	
17	Furnish and place concrete with ties	90	CY	\$400.00	\$36,000.00	
18	Place and compact embankment	960	CY	\$4.00	\$3,840.00	
SADDLE DAM						
19	Earthwork and site preparation	2,750	CY	\$4.00	\$11,000.00	
20	Core Wall inspection	470	CY	\$6.00	\$2,820.00	
21	Remove core wall to elevation 99	12	CY	\$100.00	\$1,200.00	
22	Remove spillway	42	CY	\$100.00	\$4,200.00	
23	Membrane lining	1,325	SY	\$3.00	\$3,975.00	*
24	Filter blanket	985	CY	\$5.00	\$4,925.00	*
25	Class 3 riprap	1,480	CY	\$25.00	\$37,000.00	*
26	Class 2 riprap	315	CY	\$20.00	\$6,300.00	

	SUBTOTAL				\$539,640.00	*
	MOBILIZATION AT PERCENT			5	\$18,609.50	* Mobilization items 2 to 26
	CONTINGENCIES AT PERCENT			20	\$107,928.00	*

	CONSTRUCTION COST				\$666,177.50	*
	ENGINEERING AT PERCENT			8	\$53,294.20	*
	GEOTECHNICAL / CONSTRUCTION MANAGEMENT SERVICES				\$40,000.00	*

	TOTAL ESTIMATED PROJECT COST			December 1985	\$759,471.70	*
				December 1986	\$782,255.85	* w/3% inflation
				December 1987	\$829,191.20	* w/6% inflation

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APPENDIX

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REPORT
OF
GEOTECHNICAL INVESTIGATION

PHASE II - CHESSMAN DAM
HELENA, MONTANA

TO
MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS
HELENA, MONTANA

PREPARED
BY
NORTHERN ENGINEERING AND TESTING, INC.
CONSULTING GEOTECHNICAL ENGINEERS
GREAT FALLS, MONTANA

NOVEMBER, 1985

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INTRODUCTION

General

Chessman Dam, located about 12 miles southwest of Helena, Montana, was constructed in the early 1900's. It is an earthfill dam, approximately 40 feet high, with a concrete cutoff wall extending through the embankment. Due to the age of the structure, and residential development downstream, there has been concern regarding the safety of the dam. Investigations were conducted in 1978 and 1983, to determine if the structure meets current safety standards, and what measures would be necessary to upgrade it. Investigations conducted in 1983 identified several problems with the dam, including:

- 1) the upstream and downstream slopes do not satisfy recommended stability criteria,
- 2) there is a layer of loose sand beneath the south portion of the embankment, which would be subject to liquefaction if an earthquake occurs,
- 3) the concrete cutoff wall does not extend through the foundation soil to bedrock,
- 4) there is excess pore water pressure in the foundation soil near the toe of the dam. Our report dated December 1, 1983, describes the structure, each of the deficiencies, and alternates for rehabilitation, in detail.

Due to the high potential safety hazard, the reservoir was lowered and maintained at a low level during the summers of 1984 and 1985. It was recently decided to perform additional investigations, sufficient to complete the rehabilitation design and to facilitate development of an accurate construction cost estimate.

Scope of Services

The scope of services for this investigation was outlined in our proposal dated July 22, 1985, and is summarized below:

1. Define the extent of the loose sand layer below the embankment by performing test borings on the upstream and downstream face of the dam. Sufficient investigation was to be conducted to determine the relative feasibility of Alternates 1 and 2, as they were discussed in our report of December 1, 1983.
2. Conduct an investigation to identify a borrow source with a minimum of 20,000 cubic yards of common embankment borrow, and a suitable source of rip rap.
3. Perform the necessary analysis and provide a report with recommendations so that design and cost estimates for the most attractive rehabilitation alternate could be prepared.

FIELD INVESTIGATIONS

Field investigation during this phase were conducted utilizing a truck-mounted drill, and a tractor-mounted backhoe. Thirteen test borings were made to depths of about 14 to 44 feet, in the embankment area. Seventeen test pits were excavated to depths of about 7 to 10 feet in the potential borrow area. The locations of borings and test pits made during this phase, and during previous investigations, are shown on the enclosed revised Drawing No. 83-575-1. Boring locations and elevations were provided by your surveyors.

The borings and test pits were logged by our geologist. Continuous logs of the soil conditions were recorded, standard penetration resistance tests performed, samples obtained, and groundwater levels measured, during the field investigations.

LABORATORY INVESTIGATIONS

Samples obtained during the field investigation were taken to the laboratory where they were carefully inspected and visually classified in accordance with the Unified Soils Classification System. Representative samples were then selected for physical and engineering properties tests. These included grain-size distribution, Atterberg limits, moisture-density relationship, and triaxial shear tests.

Results of all field and laboratory tests are summarized on the enclosed Tables and Plates. This information, along with the field observations, was used to prepare the final test boring and test pit logs shown in the Appendix. We have also included information from our previous investigation.

FINDINGS

The embankment condition, area geology, and subsurface material properties, were discussed in detail in our previous report. Additional findings from the field investigations conducted during this phase are discussed below.

Embankment Area

Additional test borings made on the upstream slope of the embankment indicate the loose sand layer is more extensive in this area than previously anticipated. The approximate revised limits of the loose sand layer are shown on the Drawing. The elevation of the bottom of the loose layer is somewhat variable, but based on the test borings, it appears that overexcavation to approximate elevation 60, within the limits shown, should remove the layer.

Borrow Area

An area in the northeast corner of the reservoir was investigated as a borrow source. The location of the borrow area, and the test pits, is shown on the Drawing. In this area, clayey or silty sand overlies granite bedrock. The depth to the bedrock ranges from about 7 to more than 10 feet. It appears there is well in excess of 20,000 cubic yards of sand available in this area.

Laboratory tests indicate the sand has physical and engineering properties similar to the material in the existing embankment. Triaxial shear tests indicated an angle of internal friction of about 36 degrees for remolded samples of the sand.

Water was encountered in many of the test pits, and the in-place moisture content of the sand is above optimum moisture. Unless the reservoir level is lowered more, it will probably be necessary to dry the borrow, in order to place and compact it.

Rip Rap Source

The existing rip rap on the upstream slope of the dam is in good condition and appears to have performed well. It consists of near cubical shaped blocks, and was apparently placed by hand to achieve a smooth face. The existing rip rap can probably be removed and reused.

Two additional sources of rip rap were investigated. The most promising source appears to be at an old mine adit, approximately 2000 feet northwest of the dam. (See Drawing) There are several piles of quartz monzonite boulders near the mine adit. Most of the boulders are approximately 1 to 1 1/2 feet in diameter, and do not appear to have weathered appreciably. This may be the source of the rip rap used in the original construction.

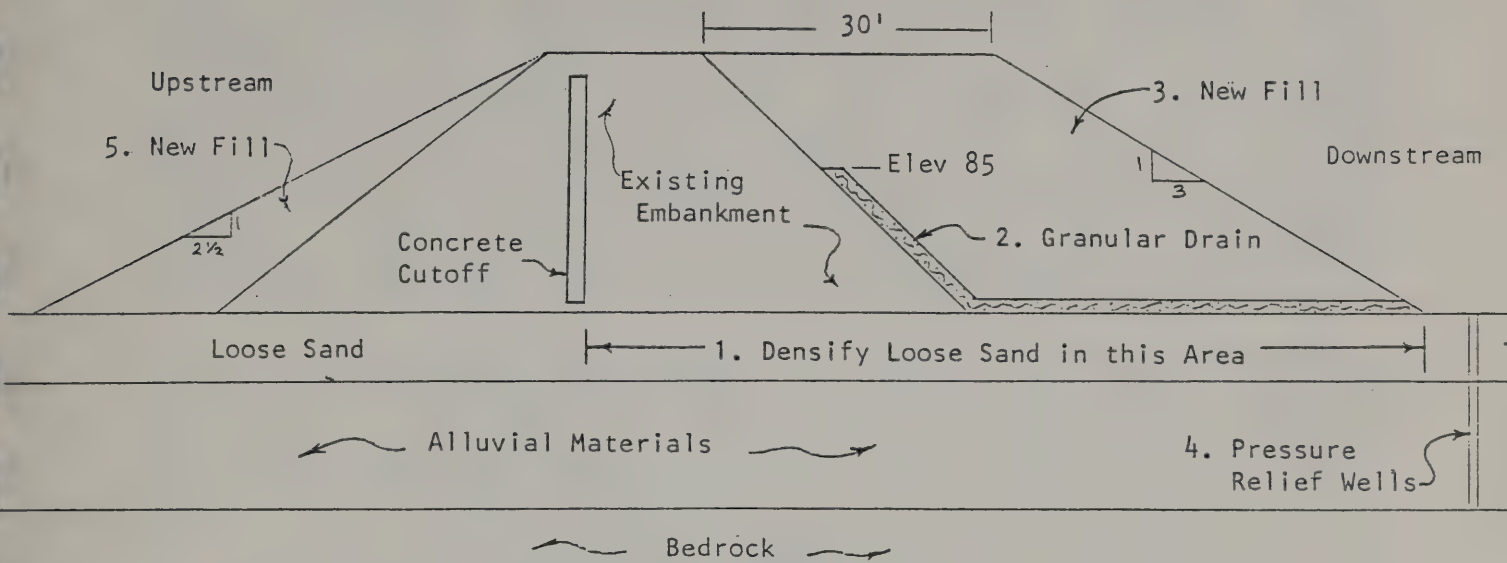
A talus slope, approximately one mile southwest of the dam was also investigated. Material in the slope consists of angular rhyolite particles, ranging in size from 2 inches to about 2 feet. There is a large quantity of the talus present, but visual observation indicates it may be subject to weathering. If this source is considered, durability tests should be conducted in the laboratory to determine if the quality is acceptable.

DISCUSSION

Recommended Rehabilitation

Alternates for rehabilitation were discussed in our previous report. Alternate 1 involved densifying the loose sand layer beneath the entire embankment. Since the loose layer is more extensive than anticipated beneath the upstream portion of the embankment, this alternate does not now appear feasible.

Alternate 2, which is recommended, involves densifying the loose sand layer downstream of the concrete cutoff. The crest of the embankment would be widened by 30 feet. With this system, if the sand layer upstream of the cutoff liquefies and causes embankment failure, there would still be sufficient embankment present to prevent loss of the reservoir. As part of this alternate, the upstream and downstream slopes of the embankment would be flattened, an internal drain would be constructed in the downstream embankment, and pressure relief wells would be installed near the toe of the embankment. The steps required are shown on the diagram on the following page, and are described below the diagram.



1. Densify the loose sand layer downstream of the concrete cutoff by overexcavating and replacing it where possible, and densifying it in-place, immediately downstream of the cutoff, (see diagram below for identification of items).
2. Construct a granular drain on the downstream slope.
3. Place fill to widen the crest by 30 feet, and slope the downstream slope at 3 horizontal to 1 vertical.
4. Install pressure relief wells around the toe of the embankment.
5. Add fill and flatten the upstream slope to 2 1/2 horizontal to 1 vertical.

Design Considerations

One of the most critical items will be the method selected to densify the loose sand layer. Where possible, overexcavation and replacement with compacted fill is probably the most economical. This can be performed downstream of the existing embankment, but construction excavations would probably undercut the concrete cutoff, causing settlement and damage if the loose sand layer is excavated immediately downstream. There is a similar problem where the loose sand is present below the existing outlet works. For this reason, a method of in-place densification, which does not induce settlement, will probably be required in these areas.

In-place densification methods which were considered included dynamic compaction, vibroflotation, and compaction grouting. Dynamic compaction involves dropping a large weight on the ground, and achieves densification through impact loading and vibration. Dynamic compaction would cause vibration and settlement, which could cause damage to the concrete cutoff and outlet works. For these reasons, it does not appear feasible. Vibroflotation could cause similar damage to these structures, and the soils appear to have too high a clay content for vibroflotation to be effective. Compaction grouting appears to be the most favorable means of densifying the sand layer. Approximate costs have been provided in the Recommendations, although specialty contractors should be contacted for more accurate estimates.

The granular drain constructed between the existing embankment and the new fill downstream should consist of a clean, free draining aggregate, and should be protected from contamination and plugging with a geotextile filter, or a graded aggregate filter. The geotextile will be less expensive and easier to construct. Design criteria are provided in the Recommendations.

The upstream slope of the embankment should be flattened, in order to be stable under sudden reservoir drawdown conditions. The use of a berm instead of slope flattening was considered, but does not appear to be feasible.

Rip rap will be necessary on the upstream slope to prevent erosion from wave action. Procedures are available for sizing the rip rap, based on wind velocity, fetch distance, and wave height. Depending upon the rip rap gradation, it may be necessary to provide a gravel layer between the embankment slope and the rip rap to prevent washing out of the fine material through the rip rap. It may be more economical to provide a geotextile between the rip rap and the embankment. Preliminary guidelines are provided in the Recommendations for sizing rip rap.

Excess hydrostatic pressure in the foundation soils beneath the toe of the dam should be relieved. Construction of a toe drain was considered, but installation of relief wells will probably be more economical.

RECOMMENDATIONS

The following corrective items should be implemented.

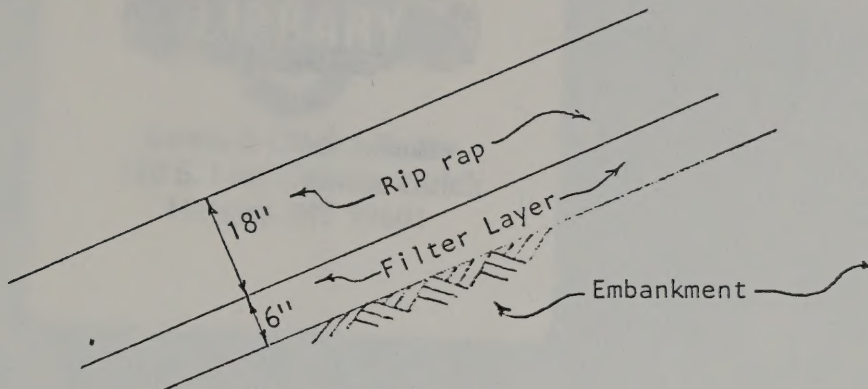
1. Densify the loose sand layer downstream of the concrete cutoff by overexcavation and replacement, or by compaction grouting. In place materials should be densified above elevation 60, within the zone shown on the Drawing.
2. For planning purposes, construction excavations should be no steeper than 1 horizontal to 1 vertical above the water table, or 2 horizontal to 1 vertical below the water table, or be shored.
3. Fill should be placed in uniform lifts and compacted to a minimum of 95 percent of maximum dry density, as determined by ASTM D698. All borrow should be approved by our geotechnical engineer. Clayey or silty sand fill is available from the borrow area in the northeast area of the reservoir.

4. Compaction grouting, or other suitable means of in-place densification, will be required immediately downstream of the concrete cutoff wall and beneath the outlet works. A qualified grouting contractor experienced in this technique should be consulted for an accurate cost estimate. For preliminary estimating purposes, assume the following:
 - A. Grout holes will be required on about 10 foot centers over the area to be treated.
 - B. Grout holes should extend to about elevation 60.
 - C. Approximately one cubic yard of the grout should be injected into each hole.
 - D. Preliminary cost estimates may be determined, assuming about \$10.00 per foot for drilling, and \$200.00 per cubic yard for the grout in place.
5. The crest of the dam should be widened by 30 feet, and the downstream slope flattened to 3 horizontal to 1 vertical.
6. Provide a granular drain between the existing embankment and the new fill, as shown in the Discussion.
 - A. The drain should be a minimum of 2 feet thick, perpendicular to the slope, should extend upward to elevation 85, and should be allowed to drain freely at the toe.
 - B. Granular drain material should meet the following gradation requirements.

<u>Screen or Sieve Size</u>	<u>Percent Passing</u>
1 inch	100
3/4 inch	85 - 95
1/2 inch	40 - 75
3/8 inch	20 - 55
No. 4	0 - 10

- C. Provide a nonwoven geotextile filter on both sides of the granular drain. A fabric similar to Mirafi 140 N, or an approved equivalent, should be specified.
7. The upstream slope of the dam should be flattened with engineered fill to at least 2 1/2 horizontal to 1 vertical.
8. Provide pressure relief wells around the toe of the embankment. The wells should be a minimum of 4 inches in diameter, spaced on 30 foot centers, extend at least 20 feet deep, and extend around the toe to elevation 75 on either abutment.

9. For estimating purposes, the following rip rap section should be provided on the upstream slope.



- A. Rip rap should conform to the following requirements:

<u>Weight of Stove</u>	<u>Percent Smaller than Given Weight</u>
100 lb	100
60 lb	90 - 70
25 lb	60 - 40
2 lb	0 - 10

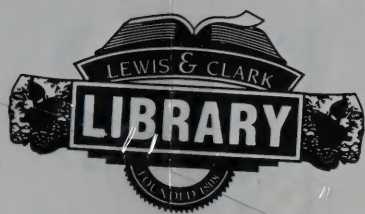
- B. Gravel for the filter layer should be well graded, and less than 3 inches in size.
- C. As an alternate, a geotextile with suitable protective covering may be considered in lieu of the gravel layer.
- D. The size of the rip rap shown above should be reviewed, after maximum wave heights have been determined.
10. Design details should be reviewed by our geotechnical engineering personnel, as they are developed.

The conclusions reached and recommendations provided in this report, are based on the assumptions stated, along with interpolation of subsurface conditions between boring locations. If construction reveals conditions different from that assumed in our analysis, it may be necessary to modify recommendations.

If changes in the nature, design, or location of the dam are planned, the recommendations contained in this report shall not be considered valid unless the changes are reviewed and the recommendations of this report modified or verified in writing.

Respectfully submitted,

Al Stilley, P.E.
Project Engineer



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